CLEVELAND'S INNERBELT BRIDGE

FORM DQP 2.01-1
LEVEL 1 CHECK PRINT SIGN-OFF SHEET

Client Name: **Ohio Department of Transportation**

Job Title: **Cleveland Innerbelt Design-Build Contract**

Job Number: **CUY-90-14.90**

Document Title: INSPECTION WALKWAY SUPPORT BEAM & HANGER SYSTEM - RFI CHANGE

Check Level (Mark One):

- [X] 1A 100% Document Check
- [ ] 1B 100% Input Check

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Form DQP 2.01-1

DJG 10/20/11
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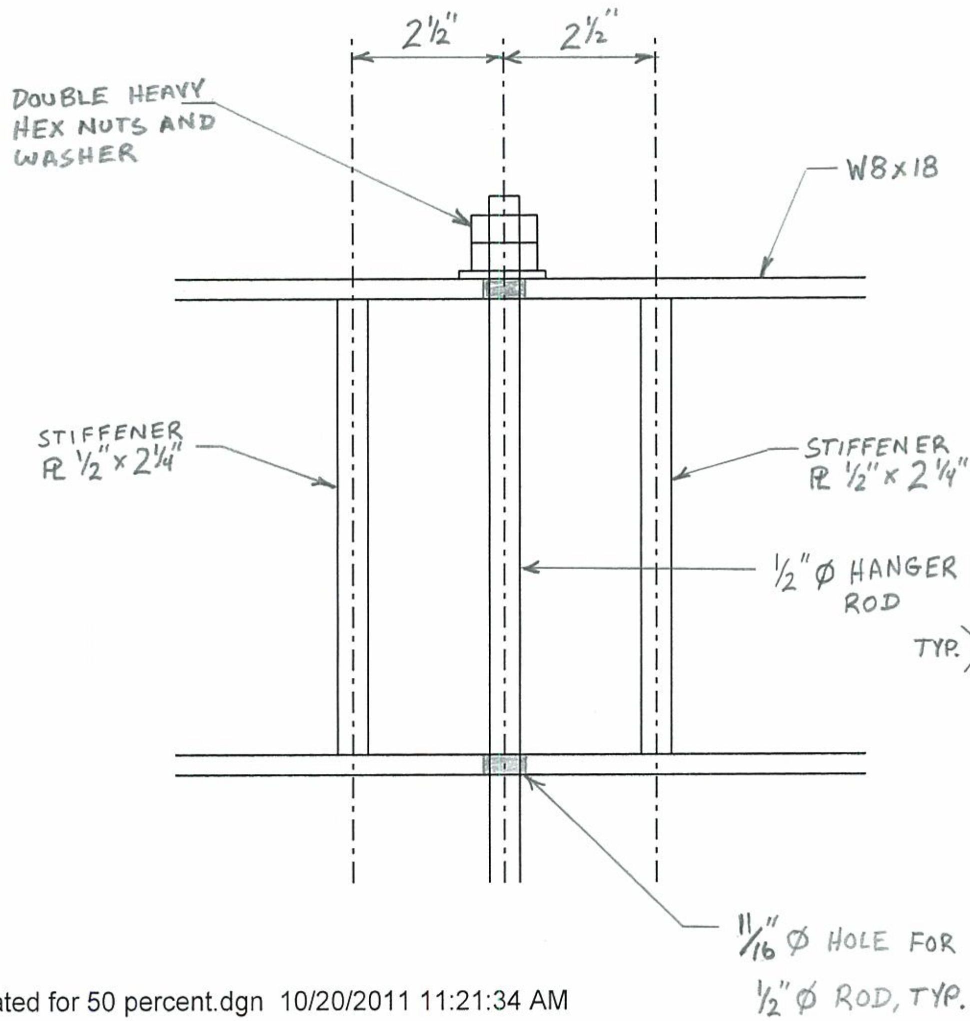

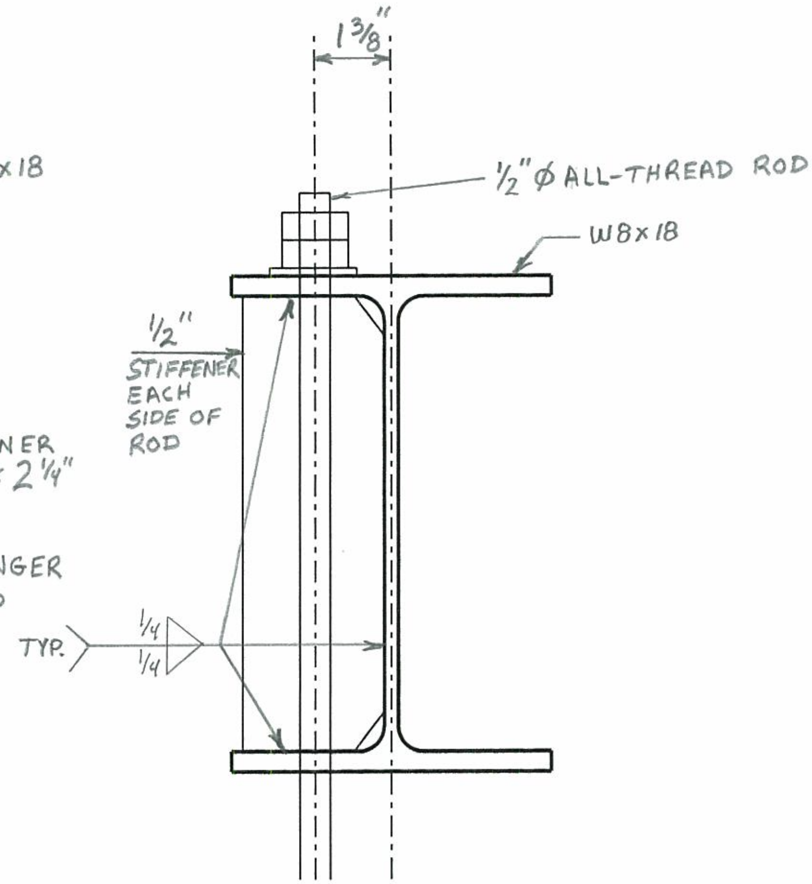

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DJG 10/20/11
SJL 10-20-11
DJG 10/20/11

RFI-013 -- Investigate replacing the C7 x 14.75 with a W8 x 18

From Channel Design Calculation by LJD 8/2/11

$$P := 2963\text{lbf}$$

$$w := 18\frac{\text{lbf}}{\text{ft}}\cdot 1.25$$

$$R_B := w\cdot\frac{15\text{ft}}{2} + \left(\frac{14.42\text{ft}}{15\text{ft}}\cdot P\right) + \left(\frac{10.583\text{ft}}{15\text{ft}}\cdot P\right)$$

$$R_B = 5.108\cdot\text{kip}$$

$$R_A := (2\cdot P) + (w\cdot 15\text{ft}) - R_B$$

$$R_A = 1.156\cdot\text{kip}$$

$$M_{max} := \frac{R_A + \left[R_A - w\cdot(15\text{ft} - 4.4167\text{ft})\right]}{2}\cdot(15\text{ft} - 4.4167\text{ft})$$

$$M_{max} = 10.972\cdot\text{ft}\cdot\text{kip}$$

From AISC Manual of Steel Construction 13th Edition Pg. 3-131

$$\phi Mn_b := 34.5\text{ft}\cdot\text{kip}$$

$$\boxed{Check_{bend} := \text{if}\left(M_{max} \le \phi Mn_b, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$$

Since Hangers are eccentric to the shear center, consider torsional effects

See attached example from *Steel Structures Design and Behavior.* Salmon, Charles and Johnson, John. Third Edition

$$e_{hanger} := 1.375\text{in}$$

$$T := 2P\cdot e_{hanger} \qquad T = 0.679\cdot\text{kip}\cdot\text{ft}$$

$$E := 29000\text{ksi} \qquad C_w := 122\text{in}^6 \qquad G := 0.385E \qquad J := 0.172\text{in}^4$$

Torsional Reaction

$$T_2 := T \cdot \left(\frac{12.5}{15} \right) \qquad T_2 = 0.566 \cdot ft \cdot kip$$

$$P_H := \frac{T}{7.81in} \qquad P_H = 1.043 \cdot kip$$

$$V_f := \frac{P_H \cdot 12.5ft}{15ft} \qquad V_f = 0.869 \cdot kip$$

The lateral bending moment is then:

$$M_f := V_f \cdot 2.5ft \qquad M_f = 2.174 \cdot ft \cdot kip$$

acting on one flange. Twice the moment acting on the entire section gives:

$$S_y := 3.04in^3 \qquad f_{bw} := \frac{2 \cdot M_f}{S_y}$$

$$f_{bw} = 17.16 \cdot ksi$$

For torsional shear stress, since $M_z := \frac{T}{2} = 0.34 \cdot kip \cdot ft$

flange thickness = $t_f := 0.33in$ $\qquad \upsilon_{sf} := \frac{M_z \cdot t_f}{J} = 7.817 \cdot ksi$ Flange

web thickness = $t_w := 0.230in$ $\qquad \upsilon_{sw} := \upsilon_{sf} \cdot \frac{t_w}{t_f} = 5.448 \cdot ksi$

For lateral bending flange shear stress:

$$Q_f := \frac{5.25in}{2} \cdot 0.330in \cdot \frac{5.25in}{4} \qquad Q_f = 1.137 \cdot in^3$$

$$I_f := \frac{1}{12} \cdot 0.330in \cdot (5.25in)^3 \qquad I_f = 3.979 \cdot in^4$$

$$\upsilon_f := \frac{V_f \cdot Q_f}{I_f \cdot t_f} \qquad \upsilon_f = 0.753 \cdot ksi$$

Stress on the entire section from normal bending:

$$\sigma_b := \frac{M_{max}}{15.2in^3} = 8.662 \cdot ksi$$

Total normal stress = $\sigma_b + f_{bw} = 25.822 \cdot ksi$

Web shear stress =

$$Q_w := 5.25in \cdot 0.330in \cdot \left(\frac{8.14in - 0.33in}{2}\right) + \left(\frac{8.14in - 2 \cdot 0.33in}{2}\right) \cdot 0.23in \cdot \left(\frac{8.14in - 2 \cdot 0.33in}{4}\right)$$

$$Q_w = 8.374 \cdot in^3$$

$$\upsilon := \frac{R_B \cdot Q_w}{61.9in^4 \cdot t_w} + \upsilon_{sw} = 8.452 \cdot ksi$$

Flange shear stress =

$$Q := \frac{(5.25in - 0.23in)}{2} \cdot 0.330in \cdot \frac{(8.14in - 0.33in)}{2} = 3.235 \cdot in^3$$

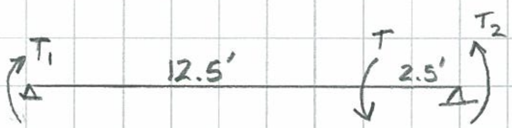
$$\upsilon := \frac{R_B \cdot Q}{61.9in^4 \cdot 0.33in} \qquad \upsilon = 0.809 \cdot ksi$$

$$\upsilon + \upsilon_{sf} + \upsilon_f = 9.378 \cdot ksi$$

In all cases stresses due to torsion and bending are considerably less than fy of 50 ksi

For	Cleveland Innerbelt	Job no.	49633	Sheet no.	
Made by	DJG	Checked by	SJL	Backchecked by	DJG
Date	9/16/11	Date	9-16-11	Date	9/16/11

HNTB



CONSIDER T to act @ 2.5' Away From Right Support.

$$T_2 = T\left(\frac{12.5}{15}\right)$$

$$T_2 = 0.833T$$

$$T = 2(2.963^{k})(1.25in)$$

$$T = 7.4 \text{ in-K}$$

ME101-1204

LRFD BEARING STIFFENER DESIGN

Bridge: **Cleveland Innerbelt - Unit 2**

Location: **Stiffeners for Catwalk Hanger Beam**

AASHTO 2010 LRFD Bridge Design Specifications (6.10.11.2)

Girder Properties

Bottom Flange Width, b_{bf} = **5.25** in

Web Depth, D = **7.48** in

Web Thickness, t_w = **0.230** in

Bearing Stiffener Properties

Stiffener Plate Width, b_t = **2.25** in

Stiffener Plate Thickness, t_p = **0.250** in

Stiffener Yield Stress, F_{ys} = **50** ksi

Modulus of Elasticity, E = **29,000** ksi

Factored Design Reactions

Factored Dead Load = **3.0** kips

Includes Live Load

Factored HL-93 Load Plus IM= **0.0** kips

Total Reaction = **3.0** kips

Projecting Width (6.10.11.2.2)

Avail Plate Width, ½ (min(b_{bf}) - t_w)/cos(skew) = **2.51** in (available)

Skew = **0.0000**

Max Plate Width = **2.89** in (6.10.11.2.2-1)

Minimum Plate Thickness, t_{min} = **0.195** in (6.10.11.2.2-1)

Minimum Plate Thickness Exceeded - O.K.

Plate Width Less Than Maximum - O.K.

Bearing Resistance (6.10.11.2.3)

Bearing Resistance Factor, Φ_b = **1.00** (6.5.4.2)

Bearing Area, A = **0.63** in^2 (conservative)

(bearing area assumes 2 plates, each with one 1 inch clip)

Bearing Resistance, B_r = **44** kips (6.10.11.2.3-2)

Bearing Resistance Exceeds Total Reaction - O.K.

Axial Resistance (Column) (6.10.11.2.4)

Comp. Resistance Factor, Φ_c = **0.90** (6.5.4.2)

Column Area = **1.13** in^2

Column Moment of Inertia = **0.64** in^4

Radius of Gyration, r = **0.75** in

K = **0.75** (6.10.11.2.4a)

Po = **56.25** kip

Pe = **5779.84** kip (6.9.4.1-3)

Pe/Po = **102.75**

Axial Resistance, P_r = **50.42** kips

Axial Resistance Exceeds Total Reaction - O.K.

Fillet Weld Thickness (6.13.3.2.3b)

Total Reaction = **3.0** kips

Minimum Weld Size = **0.25** in.

Weld Size req'd by stress = **0.0213** in.

Use Double Fillet weld = **0.25** in. **= 1/4" weld**

Design Summary

Material: **A709 Grade 50**

Plates: **2 - 0.250 in x 2.25 in x 7.48 in**

■ **EXAMPLE 8.6.1**

Compute the stresses on the W18×71 beam of Example 8.5.2 and Fig. 8.5.8 using the flexural analogy rather than the differential equation solution.

SOLUTION

The substitute system is as shown in Fig. 8.6.2a. The lateral bending moment is then

$$M_f = V_f(L/2) = 1.13(12) = 13.6 \text{ ft-kips}$$

acting on one flange. Twice the moment acting on the entire section gives

$$f_{bw} = \frac{2M_f}{S_y} = \frac{2(13.6)(12)}{15.8} = 20.6 \text{ ksi}$$

For torsional shear stress, since $M_z = T/2 = 20$ in.-kips,

$$v_s = \frac{M_z t}{J} = \frac{20(0.810)}{3.39} = 4.78 \text{ ksi} \quad \text{(flange)}$$

$$v_s = 4.78\left(\frac{0.495}{0.810}\right) = 2.92 \text{ ksi} \quad \text{(web)}$$

For lateral bending flange shear stress,

$$v_w = \frac{V_f Q_f}{I_f t_f} = \frac{1.13(5.90)}{(30.0)0.810} = 0.27 \text{ ksi}$$

where $Q_f = (7.635/2)(0.810)(7.635/4) = 5.90$ in.^{3} ■

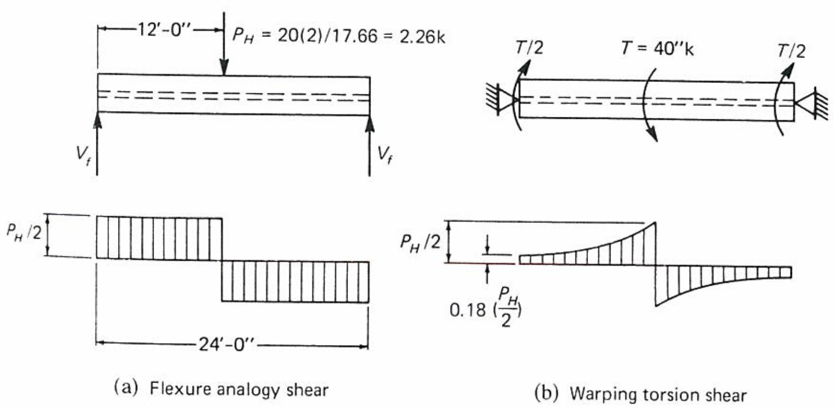


Figure 8.6.2 Comparison of lateral shear on flange due to warping torsion with that from simple lateral flexure analogy.

The results of the two methods are compared as follows:

Type of Stress	Flexural Analogy	Differential Equation
Normal stress $= f_b + f_{bw} = 11.3 + 20.6 =$	31.9 ksi	19.83 ksi
Web shear stress $= v + v_s = 1.25 + 2.92 =$	4.17 ksi	3.65 ksi
Flange shear stress $= v + v_s + v_w = 0.27 + 4.78 + 0.27 =$	5.32 ksi	4.24 ksi

It is apparent that use of the flexure analogy without modification is a very conservative approach. In some situations it is so excessively conservative as to be practically useless. Furthermore, the most important design item, the lateral bending normal stress f_{bw} is overestimated by the greatest amount.

The relationship between the flexural analogy and the true torsion problem is best illustrated by referring to Fig. 8.5.4a. Note that the full torsional shear resulting from M_s and M_w is analogous to the lateral flexure problem. Figure 8.5.4b shows the portion of the shear that goes into rotation of elements, while Fig. 8.5.4c shows the portion contributing to lateral flange bending. If one could correctly assess how the shear due to warping torsion compares with the lateral flexure situation, design for torsion could be greatly simplified without being grossly conservative.

Figure 8.6.2b shows the accurate variation of V_f for the problem of Example 8.6.1, computed according to Eq. 8.5.7, whereupon

$$V_f = \frac{T}{2h}\left(\frac{\cosh \lambda z}{\cosh \lambda L/2}\right) \tag{8.6.1}$$

in which the shear from the lateral bending analogy, $T/2h$, is modified by the hyperbolic function.

The lateral bending moment can thus be expressed for this problem as

$$M_f = \beta \frac{T}{2h}\left(\frac{L}{2}\right) \tag{8.6.2}$$

or, in general, the change in lateral moment between the support and location of zero shear is

$$\Delta M_f = \beta \times (\text{area under flexure analogy shear diagram}) \tag{8.6.3}$$

where β is a reduction factor that depends on λL.

It is to be noted that if Eq. 8.6.2 is multiplied by h, and the concentrated moment T is thought of as a concentrated load, the analogous moment $M_f h$ (sometimes referred to as *bimoment*) equals β times the simple beam moment.

Thus the modified flexure analc

1

for the case of Fig. 8.6.1.

Tables 8.6.1 through 8.6.5 g loading and restraint conditions. equals $M_f h$ above) or the curv may be used. In Tables 8.6.3 an unit length (say, in.-kips/ft).

■ **EXAMPLE 8.6.2**

Recompute the stresses due to tc modified flexural analogy methc

SOLUTION

The flexure analogy gives

M

as previously computed.

$\lambda L = 4.80$ (as

From Table 8.6.1 at $a = 0.$ flexure analogy value. Thus the

$M_f = 13.6($

$f_{bw} = \frac{2M_f}{S_y}$

which compares favorably with equation solution using $\lambda L = 4.$ the β modified flexure analog differential equation solution val

8.7 PRACTICAL SITUATIO

There are relatively few occasioi can cause significant twisting, construction. In most building strained by attachments along th not free to twist. Even though tc because the rotation cannot exc members.

Thus the modified flexure analogy gives

$$M_f h = \beta\left(\frac{TL}{4}\right) \tag{8.6.4}$$

for the case of Fig. 8.6.1.

Tables 8.6.1 through 8.6.5 give "exact" values for β for several common loading and restraint conditions. For other cases Table I of Ref. 8.9 (where M_w equals $M_f h$ above) or the curves of *Torsion Analysis of Steel Members* [8.8] may be used. In Tables 8.6.3 and 8.6.4, m is the applied torsional loading per unit length (say, in.-kips/ft).

■ EXAMPLE 8.6.2

Recompute the stresses due to torsion on the beam of Example 8.6.1, using the modified flexural analogy method utilizing the β values from Table 8.6.1.

SOLUTION

The flexure analogy gives

$$M_f = 13.6 \text{ ft-kips}$$

as previously computed.

$$\lambda L = 4.80 \text{ (as computed in Example 8.5.2)}$$

From Table 8.6.1 at $a = 0.5$, $\beta \approx 0.41$, i.e., use about 41 percent of the flexure analogy value. Thus the modified flexure analogy gives

$$M_f = 13.6(0.41) = 5.58 \text{ ft-kips}$$

$$f_{bw} = \frac{2M_f}{S_y} = \frac{2(5.58)12}{15.8} = 8.48 \text{ ksi}$$

which compares favorably with $f_{bw} = 8.49$ ksi as computed by the differential equation solution using $\lambda L = 4.80$. For this case that exactly fits a table case, the β modified flexure analogy is the "exact" value obtained from the differential equation solution value. ■

8.7 PRACTICAL SITUATIONS OF TORSIONAL LOADING

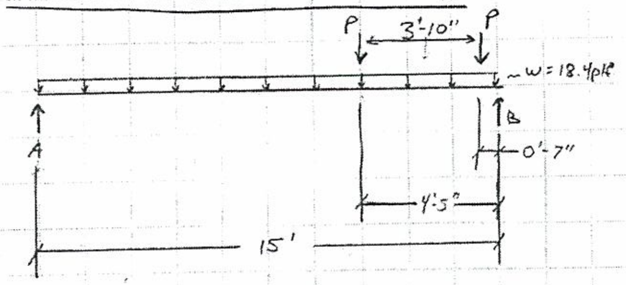
There are relatively few occasions in actual practice where the torsional load can cause significant twisting, and frequently these situations arise during construction. In most building construction the members are laterally restrained by attachments along the length of the member and therefore they are not free to twist. Even though torsional loading exists, it may be self-limiting because the rotation cannot exceed the end slope of the transverse attached members.

The following pages are excerpts from calculations posted in our final design submittal and are not given in their entirety. The pages presented are shown to verify loads used in the previous hanger design for use in the revised design. For the entire calculation please refer to the original calculations submitted with the Unit 2 Structural Steel Final Design Package.

For CATWALK CHECK/REDESIGN	Job no. 49683	Sheet no. 21
Made by LJD	Checked by KDG	Backchecked by LJD
Date 8/2/11	Date 8-4-11	Date 8/5/11

HNTB

SUPPORT CHANNEL (CONT'D)



$P = 2469 \times 1.2 = 2963\#$

$w = 18.4\ \#/ft$ (From Page 10 of 3/3/11 calcs)

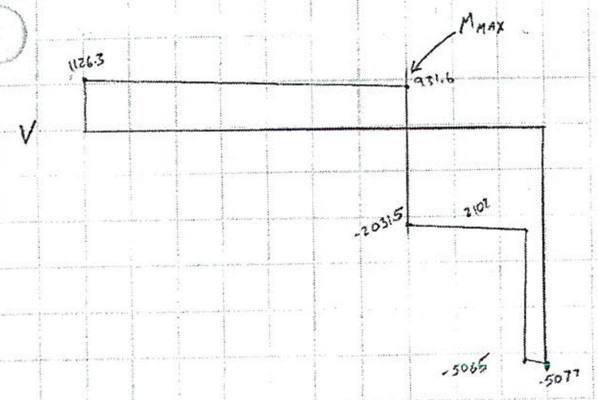
$R_B = \left(18.4\#/ft \times \frac{15'}{2}\right) + \left(\frac{14.42'}{15'} \times 2963\#\right) + \left(\frac{10.583'}{15'} \times 2963\right)$

$= 138\# + 2848.5\# + 2090.5\#$

$= 5077\#$

$R_A = (2 \times 2963\#) + (18.4\#/ft \times 15') - 5077\#$

$= 1126.3\#$



$M_{MAX} = \left(\frac{1126.3 + 931.6}{2}\right) \times 10.583 = 10890\ \#\text{-ft}$

$M_u = 10890\ \text{ft-lb} < \phi M_n = 14250\ \text{ft-lb}$, OK

(SEE PAGE 11 OF 3/3/11 CALCS AND PAGE 13, REVISION COMPLETED BY KDG AND MX ON 5/3/11.)

CHANNEL TO GIRDER, CHANNEL TO STRINGER CONNECTION

. ORIGINAL DESIGN OF THIS CONNECTION (SEE 3/15/11 CALCS COMPLETED BY AMS, ATTACHED) CALCULATED BOLTED CONNECTION FORCES AS IF IT WERE A FIXED-FIXED BEAM.

THIS SHOULD HAVE BEEN DESIGNED AS A PINNED END, SIMPLY SUPPORTED BEAM. BY INVESTIGATION, ALL COMPONENTS ARE CONSERTIVELY DESIGNED AND WILL BE OK FOR ADDITIONAL LOAD.

GRIP STRUT

MAXIMUM FLOORBEAM SPACING = 30'. SO, MAXIMUM WALKWAY SUPPORT SPAN ≈ 15'. FOR 9 GAUGE GRIP STRUT, MAXIMUM CLEAR SPAN FOR 85 PSF IS 14'. NEED TO USE HSS 12X2X1/4 TO REDUCE CLEAR SPAN TO 14'.

UPDATE PLANS ACCORDINGLY.

1204

The following 22 pages are for reference for the preceding check/redesign of the catwalk located at the top of the girders. For efficiency, the design completed by DMS on 3/3/2011 (the following calculations) will be used except for details that must be updated due to the change in floorbeam layout.

ALLOWABLE DESIGN Loads/Deflection "H" Series

WALKWAYS

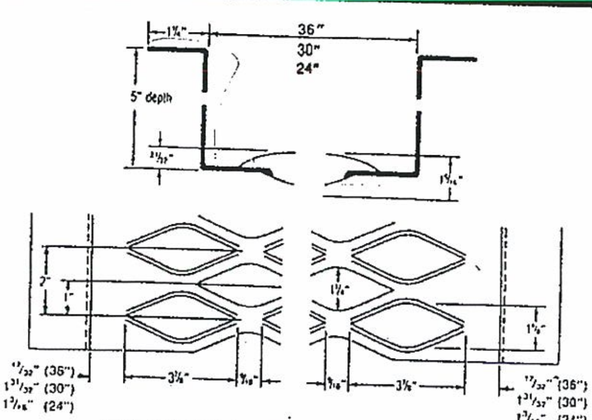
5-diamond (24″ wide)
6-diamond (30″ wide)
8-diamond (36″ wide)

STEEL WALKWAY SELECTION & DESIGN LOAD/DEFLECTIONS

Walkway						Clear Span																
Width	Thickness Gauge	Channel Depth in.	Weight lb./ft.	Catalog Number	Load Type	4′-0″	5′-0″	6′-0″	7′-0″	8′-0″	9′-0″	10′-0″	11′-0″	12′-0″	13′-0″	14′-0″	15′-0″	16′-0″	18′-0″	20′-0″	22′-0″	24′-0″
24″	11	5	15.4	H-55011-W	U	750	480	334	245	187	148	120	99	83	71	62	54	47	37	30	25	21
					D	0.34	0.35	0.38	0.34	0.34	0.34	0.35	0.42	0.50	0.59	0.69	0.79	0.91	1.13	1.43	1.70	2.03
					C	3000	2400	2000	1714	1500	1334	1200	1091	1000	922	857	800	750	666	600	546	500
					D	0.27	0.28	0.31	0.28	0.28	0.27	0.28	0.34	0.41	0.48	0.55	0.63	0.72	0.91	1.13	1.36	1.62
24″	10	5	17.5	H-55010-W	U	937	600	417	306	234	185	150	124	104	89	77	67	59	46	38	31	26
					D	0.38	0.39	0.42	0.38	0.38	0.38	0.39	0.47	0.56	0.66	0.77	0.88	1.01	1.26	1.59	1.89	2.25
					C	3750	3000	2500	2143	1875	1667	1500	1364	1250	1153	1071	1000	938	833	750	682	625
					D	0.30	0.31	0.34	0.31	0.30	0.30	0.31	0.38	0.45	0.53	0.61	0.70	0.80	1.01	1.25	1.51	1.80
24″	9	5	19.6	H-5509-W	U	1031	660	459	337	257	204	165	136	114	98	85	74	65	51	42	34	29
					D	0.38	0.39	0.42	0.38	0.38	0.38	0.39	0.47	0.56	0.66	0.77	0.88	1.01	1.26	1.59	1.89	2.25
					C	4125	3300	2750	2357	2063	1834	1650	1500	1375	1268	1178	1100	1032	916	825	750	688
					D	0.30	0.31	0.34	0.31	0.30	0.30	0.31	0.38	0.45	0.53	0.61	0.70	0.80	1.01	1.25	1.51	1.80
30″	11	5	17.7	H-65011-W	U	732	468	325	239	183	145	116	96	81	69	60	52	45	36	28	24	20
					D	0.33	0.39	0.36	0.36	0.41	0.38	0.37	0.37	0.44	0.51	0.59	0.68	0.77	0.98	1.20	1.46	1.73
					C	3667	2932	2444	2095	1832	1629	1467	1333	1222	1128	1047	977	916	815	732	667	610
					D	0.27	0.31	0.29	0.29	0.33	0.31	0.30	0.30	0.35	0.41	0.48	0.55	0.62	0.78	0.97	1.17	1.40
30″	10	5	19.9	H-65010-W	U	916	586	407	299	229	182	146	121	102	87	75	65	57	45	36	30	25
					D	0.37	0.43	0.40	0.40	0.46	0.42	0.41	0.41	0.49	0.57	0.66	0.75	0.86	1.09	1.33	1.62	1.92
					C	4584	3666	3056	2619	2291	2037	1834	1667	1528	1410	1309	1222	1146	1019	916	834	763
					D	0.30	0.34	0.32	0.32	0.37	0.34	0.33	0.33	0.39	0.45	0.53	0.61	0.69	0.87	1.08	1.30	1.55
30″	9	5	22.1	H-6509-W	U	1007	644	447	328	251	200	160	133	112	95	82	71	62	49	39	33	27
					D	0.37	0.43	0.40	0.40	0.46	0.42	0.41	0.41	0.49	0.57	0.66	0.75	0.86	1.09	1.33	1.62	1.92
					C	5042	4032	3361	2880	2520	2240	2017	1833	1680	1551	1439	1344	1260	1120	1007	917	839
					D	0.30	0.34	0.32	0.32	0.37	0.34	0.33	0.33	0.39	0.45	0.53	0.61	0.69	0.87	1.08	1.30	1.55
36″	11	5	20.2	H-85011-W	U	444	284	197	144	111	88	71	58	49	42	36	31	28	21	17	14	12
					D	0.35	0.35	0.30	0.29	0.30	0.32	0.35	0.38	0.46	0.54	0.62	0.71	0.82	1.04	1.26	1.50	1.78
					C	2664	2133	1777	1524	1333	1184	1066	969	888	820	761	711	666	592	533	484	444
					D	0.28	0.28	0.23	0.23	0.23	0.26	0.28	0.31	0.37	0.43	0.50	0.58	0.65	0.83	1.02	1.23	1.47
36″	10	5	22.7	H-85010-W	U	556	356	247	181	139	110	89	73	62	53	45	39	35	27	22	18	15
					D	0.39	0.39	0.33	0.32	0.33	0.36	0.39	0.42	0.51	0.60	0.69	0.79	0.91	1.15	1.40	1.67	1.98
					C	3330	2667	2222	1905	1667	1481	1333	1212	1111	1026	952	889	833	741	667	606	556
					D	0.31	0.31	0.26	0.26	0.26	0.29	0.31	0.34	0.41	0.48	0.55	0.64	0.72	0.92	1.13	1.37	1.63
36″	9	5	25.3	H-8509-W	U	611	391	271	199	152	121	97	80	68	58	49	42	38	29	24	19	16
					D	0.39	0.39	0.33	0.32	0.33	0.36	0.39	0.42	0.51	0.60	0.69	0.79	0.91	1.15	1.40	1.67	1.98
					C	3663	2933	2444	2095	1833	1629	1466	1333	1222	1128	1047	977	916	815	733	666	611
					D	0.31	0.31	0.26	0.26	0.26	0.29	0.31	0.34	0.41	0.48	0.55	0.64	0.72	0.92	1.13	1.37	1.63

ALUMINUM WALKWAY SELECTION & DESIGN LOAD/DEFLECTIONS (30″ wide)

Walkway						Clear Span																
Width	Thickness Gauge	Channel Depth in.	Weight lb./ft.	Catalog Number	Load Type	4′-0″	5′-0″	6′-0″	7′-0″	8′-0″	9′-0″	10′-0″	11′-0″	12′-0″	13′-0″	14′-0″	15′-0″	16′-0″	18′-0″	20′-0″	22′-0″	24′-0″
ALUM.	.150	5	8.4	H-650-W .150-A	U	633.00	405.00	281.00	207.00	158.00	125.00	101.00	84.00	70.00	60.00	51.60	45.00	39.50	35.00	31.20	28.00	25.30
					D	0.31	0.35	0.40	0.49	0.58	0.67	0.77	0.86	1.02	1.20	1.39	1.60	1.82	2.06	2.30	2.57	2.85
					C	3163.00	2530.00	2108.00	1807.00	1581.00	1406.00	1265.00	1150.00	1054.00	973.00	904.00	843.00	791.00	744.00	703.00	666.00	633.00
					D	0.25	0.28	0.32	0.39	0.47	0.54	0.61	0.69	0.82	0.96	1.12	1.28	1.46	1.64	1.84	2.06	2.28



ALLOWABLE DESIGN Loads/Deflections — for Heavy-Duty GRIP STRUT® Walkways/Planks: UNIFORM and CONCENTRATED Loads (U and C), corresponding siderail DEFLECTIONS (D); for individual grating struts: CONCENTRATED Loads (C_s) and corresponding strut DEFLECTIONS (D_s) — see load application details in "General Load Information", page 4.

STRUT UNIFORM/CONCENTRATED Loads/Deflections(2)

walk-way width	material thknss	UNIFORM U (lb/ft²)		CONCENTRATED C_s (lb/ft)	
		Serrated	Non-serr	Serrated	Non-serr
8-d 36″ wide steel	11 ga.	298	340	447	510
	10 ga.	343	391	515	587
	9 ga.	391	444	586	667
	DEFLEC (in)	0.20	0.19	0.16	0.15
6-d 30″ wide steel	11 ga.	429	490	537	612
	10 ga.	494	563	618	704
	9 ga.	563	640	703	800
	DEFLEC (in)	0.14	0.13	0.11	0.10
5-d 24″ wide steel	11 ga.	798	917	798	917
	10 ga.	912	1048	912	1048
	9 ga.	1026	1179	1026	1179
	DEFLEC (in)	0.11	0.10	0.08	0.07
30″ wide aluminum	.150″	590	590	679	679
	DEFLEC (in)	0.75	0.75	0.60	0.60

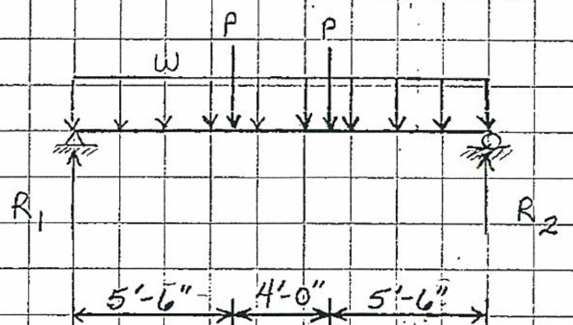
(2) See "General Load Information", page 4, for complete explanation of design load deflection conditions.

For Cleveland Innerbelt Unit 2	Job no. 49633	Sheet no. 10
Made by DMS	Checked by LJD	Backchecked by DMS
Date 2/1/11	Date 3/11/11	Date 3/15/11

HNTB

Catwalk support channel design
C7 x 14.75

Loading (Place Catwalk @ center for worst case)



(At widest stringer spacing)

$P = 2469$ lb (From Hanger Post)

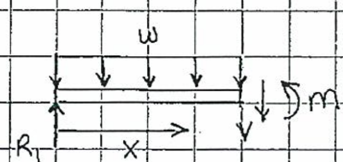
$w = (14.75)(1.25) = 18.4$ plf

↑ per C7x14.5

$\Sigma M_2 = 0 = P(5.50) + P(9.50) + w\frac{(15.0)^2}{2} - R_1(15.0)$

$R_1 = 2607.0$ lb

$R_2 = 2607.0$ lb



$m = R_1 x - \frac{wx^2}{2}$

Max M @ x = 5'-6"

$M_{max} = 2469(5.5) + \frac{18.4(15)^2}{8}$

$= 14097$ lb·ft $= M_u$

Moment Cap.

16.1 of AISC

Chap F

1. Yielding

$M_p = F_y Z = 36\text{ ksi}(9.75\text{ in}^3) = 351$ k·in
$= 29250$ lb·ft

$1.5 M_y = 1.5 F_y S = 1.5(36\text{ ksi})(7.78\text{ in}^3)$

$= 420$ k·in $= 35010$ lb·ft

$M_n = 29250$ lb·ft

2. Lateral Torsional Buckling

$L_b = 15'-0" = 180"$

$L_p = 1.76 r_y \sqrt{E/F_y} = 1.76(0.561)\sqrt{\frac{29000}{36}}$
$= 28.0"$

$r_y = 0.561"$

$S_x = 7.78\text{ in}^3$

1204

For Cleveland Innerbelt Unit 2	Job no. 49633	Sheet no. 11
Made by DMS	Checked by LJD	Backchecked by DMS
Date 2/1/11	Date 3/11/11	Date 3/15/11

HNTB

Support Channel Design Cont'd

$$M_n = F_{cr} S_x \leq M_p$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2}$$

$$c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$$

$$C_w = \frac{t_f b^3 h_o^2}{12} \left(\frac{3bt_f + 2h_o t_w}{6bt_f + h_o t_w}\right)$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{S_x h_o}{Jc}\right)^2}}$$

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}$$

$$h_o = 7 - 0.366 = 6.634 \text{ in}$$

$$C_w = \frac{0.366(2.09)^3(6.634)^2}{12} \left(\frac{3(2.09)(0.366) + 2(6.634)(0.419)}{6(2.09)(0.366) + (6.634)(0.419)}\right)$$

$$= 13.06 \text{ in}^6$$

1204

$$r_{ts}^2 = \frac{\sqrt{1.37(13.06)}}{7.78} = 0.544$$

$$r_{ts} = \sqrt{0.544} = 0.737$$

$$c = \frac{6.634}{2} \sqrt{\frac{1.37}{13.06}} = 1.07$$

$$L_r = 1.95(0.737)\left(\frac{29000}{0.7(36)}\right) \sqrt{\frac{0.267(1.07)}{7.78(6.634)}} \sqrt{1 + \sqrt{1 + 6.76\left(\left(\frac{0.7(36)}{29000}\right)\left(\frac{7.78(6.634)}{0.267(1.07)}\right)\right)^2}}$$

$$= 177.5 \text{ in}$$

$$C_b = 1.0$$

$$F_{cr} = \frac{1.0\pi^2(29000)}{\left(\frac{180}{0.737}\right)^2} \sqrt{1 + 0.078 \frac{0.267(1.07)}{7.78(6.634)} \left(\frac{180}{0.737}\right)^2}$$

$$= 24.82$$

$$M_n = 24.82(7.78) = 193.1 \text{ k-in}$$

$$= 16091.7 \text{ lb-ft}$$

$$\phi M_n = 0.9(16091.7) = 14482.5 \text{ lb-ft}$$

$$M_u = 14097 \text{ lb-ft} < \phi M_n = 14482.5 \text{ lb-ft}$$

ok

Use C7 x 14.75 for Catwalk Support Channel